ARCHED DAMS

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ARCHED DAMS

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ARCHED DAM.

INTRODUCTION.

In years past it has been the custom to design dams to resist overturning due to the water behind them by the weight of the masonry alone. The design was safe when the resultant line of pressure remained within the middle third, both for reservoir full and reservoir empty, that is, the masonry was at no portion subject to tensile stresses. Such design resulted in huge masses of masonry involving great expenditure.

This method was accepted when stresses in masonry structures were to a certain extent unknown factors in designing. However, with the advent of reinforced concrete into the engineering field, the determination of stresses became a prime factor. Such great strides were taken along those lines that we can now formulate definite ideas of the action of those materials and designing has now become the science of utilizing a little material placed to great advantage.

Of late years some attention has been called to arch dams, which are designed to resist overturning either partly or wholly by arch action. The principal idea of building this type of dam is the saving in material which is effected, over the old gravity type. It is this idea which the writers intend to affirm in this thesis.

Two distinct types of arch dam are taken up, the first being the simple arch and the second a double hollow arch dam, which may in the near future become a close competitor of the Ambursen type. The dams herein designed are proposed as acceptable alternate designs for a gravity dam which is at the present time being built by the East Creek Electric Co. on East Canada Creek, at Ing~~ra~~ham Mills, New York. The results are indeed gratifying, and are decidedly in favor of the arch dams.

DISCUSSION.

As any new idea is received by the profession at large with moore or less caution, so with the arch dam. The mathematics of the curved dams are uncertain and not reducible to a satisfactory demonstration, and consequently there is great difference of opinion among engineers as to the value of the arch in adding stability to a dam. But the practicability of this type has been proven by the Bear Valley, Zola and Sweetwater dams, the recent Las Vegas arch dams, and a large number of arch dams constructed in Australia. The Bear Valley and Zola dams are so slender as to be entirely unstable were they not supported by arch action.

The general consensus of opinion is that a curved dam will act as an arch, if the thickness of the dam does not exceed 1/3 the radius of the upstream face. The radius of the face must be at

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least 65 feet to transmit arch action as with a smaller radius the weight directly back of the dam is so great in comparison with the side thrust that the side thrust will be of very little value.

An arch dam could not be built under all conditions, as it requires natural abutments to take up the side thrust. In a case where no natural abutments existed, concrete or masonry abutments would have to be built of such great volume that it would be of no advantage.

Where the conditions are favorable, that is, the dam is to be placed across a deep, narrow valley with rock slopes, the arch dam has the following advantages:

1st. The stresses though not reducible to satisfactory demonstration are even more definite than the stresses in heavy masonry. In heavy masonry, large stresses are set up due to uneven contraction and expansion, and uneven settlement.

2nd. Curving of dams enables them to resist vertical cracks due to variations in temperature and to climatic extremes. In the case of one dam a movement of 1-1/10 inches was noticeable when the reservoir was filled and emptied. The movement due to temperature was the same being due to the sun's heat. In arched dams such movement is not harmful. Masonry contracts in cold weather, a tension of from 140 to 280 pounds being set up if the masonry is not free to contract. The curved dam takes up this movement without cracking even though the water pressure does not cause the arch to come into action. The water pressure alone tends to close such cracks.

3rd. The water pressure is perpendicular to the face at every point, and can be resolved into components perpendicular and parallel to the abutments. The first component is resisted by gravity and arch stability, while the second brings the upstream face into compression, closes all vertical cracks and consolidates the masonry transversely.

4th. If by any cause water finds its way through the foundation or between the concrete and the foundation rock, the arch dam is in not nearly so grave a danger as would be a gravity dam. In the former the water can escape more freely while it is detained under the gravity dam and exerts an upward pressure on the bottom causing the effective weight to diminish and cause it to slide or overturn.

5th. The last and most important advantage is the saving of material, which in this case amounts to about 40% in the case of the simple arch and about 50% in the hollow arch.

DESIGN OF ARCHED DAM.

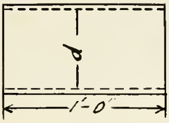
This arched dam will be designed upon the principle of stresses in pipes (thin). We will assume that the outside pressure which will cause the pipe to cave in is the same as the internal pressure which will cause the pipe to tear along the longitudinal seams.

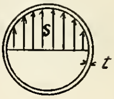
Assume a lineal foot of pipe

P = pressure tending to burst
pipe = unit pressure x diameter.
The material resisting this pressure
is s x 2t x unit of length, where t = thickness.

$$\therefore P = pd = 2ts$$

$$t = \frac{pd}{2s} = \frac{p\cancel{t}r}{s}$$





Where
t = thickness required
r = internal radius
p = total pressure acting on a unit
s = stress in material

This formula though designed for thin pipes is applicable to the dam in hand as the thickness at a section required is small in comparison with the radius of the dam.

The formula is well on the side of safety, since it requires a greater pressure to crush a pipe than to burst if from the inside, as shown by the following formula, taken from Merriman's Mechanics of Materials.

$2tS = dR$, formula for pipe under internal pressure

$R = 9,600,000\, t^{2.18}/ld$ Empirical formula for collapsing pressure per square inch, given by Wood.

Take $t = 1$ in. $d = 12"$ $l = 20'0"$ $r = 12"$

$S = 20000$

$R = \frac{20000 \times 2 \times 1}{12} = 3333$ # per Sq. in.

$R = \frac{9,600,000}{12 \times 240} = 3333$ # " " "

Thus for the conditions in hand the collapsing pressure for a 20'0" pipe is the same as the bursting pressure. But for a 10'0" the collapsing pressure is twice that of the bursting pressure.

Taking the case of the required 132'0" arch dam of radius 250'0". Ratio of radius to length = 1.895. Here the collapsing pressure would be far greater than the bursting pressure, so that we are safe in using the formula $T = \frac{RP}{S}$.

The top width of the dam was taken as 6'0" to resist not only the action of waves but of ice which in the North Eastern states is formed in thick layers.

After the values of T for the various sections were determined and plotted, the best curve to suit the profile of the back was a parabola with its axis parallel and 50 ft. away from the face. The thicknesses of the dam of this profile at various sections were determined and found to be safe.

Although various dams have been built with high compressive stress in the concrete, in this instance it was thought advisable to keep the stress below 350 lbs. per square inch. This stress would correspond closely to that in the arched dam at Las Vegas, New Mexico. This dam is 50' high and 500' long carrying a stress of 350 lbs. per square inch along the arch ring.

DESIGN OF ARCH DAM SECTION.(SINGLE ARCH.)

The formula used in this design was as stated in the discussion.

$T = RP/S$

where T = thickness in feet
R = radius of downstream face in feet
P = pressure in tons per sq.ft. on back of dam due to water pressure.
S = stress in tons per square foot.

$T = RD \times .0011$

here S = 25 tons per sq.ft.
R = 250 ft.
$T = 250 \times 10 \times .0011 = 2.75$ for 10 ft level.
$= 250 \times 20 \times .0011 = 5.50$ for 20 ft. level.

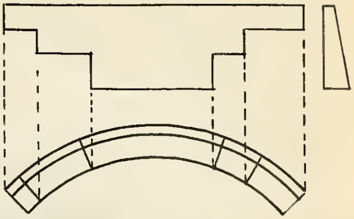
The Table 1 shows complete figure for each10 ft. level of dam.

When these thicknesses were drawn, the profile resembled a ery slender triangle. The thickness, 2.75 feet for the top was not udged sufficient to resist ice and wave action and consequently 6'0" as taken. To add 3.25 ft. at every section would be a waste of material nd a parabola would take in the increased thickness at the top without nduly increasing the bottom.

A parabola whose formula is $Y^2 = \frac{44}{3}x$ very closely approaches he desired form. The maximum ordinate is 44 and $x = 132$.

The thicknesses at the different 10 ft. level are as shown in the column for actual width on Table 1.

See Plate A for Profile.



VOLUME OF ARCH DAM.

The volume was determined as follows: At the points of stepping the point was projected down to the center line of the arch. At this point a radial line was drawn. Between two successive radii the segment has a uniform depth and the width assumed is the average width measured along the arch center line, thus determining the cross section of the segment and knowing its width the volume is obtained by multiplying the cross section by the width. This has been done in Table (

The summation of the volume of various segments gives the total volume which is found to be 17,950 cu.yds.

DESIGN OF ABUTMENTS.

The abutments carry the thrust of the upper 30'0" of the dam as between elevation 640 and 670 the material is not of sufficient crushing strength to take up the thrust.

The pressure against the abutments will be figured by the direct compression formula P = AS

where P is the total load on abutment
A = area of dam abutting
S = stress per unit of area in arch ring

The total of 30'0" will be divided into sections of 10'0".
1st section.

A - x 10 = 68.5 sq.ft.

S - 5.07 T per sq. ft.

P - AS - 68.5 x 5.07 - 347.295 tons.

This acts at a distance of 23'4" above the base.

2nd Section.

A - x 10 - 84.5

S - - 13.56

P - AS - 84.5 x 13.56 - 114.82 Tons

This acts at a distance of 14.55 above the base.

Section.

$$A = \frac{9.2 + 11.0}{2}(10) = 101.$$

$$S = \frac{16.98 + 21.29}{2} = 19.14$$

$$P = A\,S = 101 \times 1914 = 1933.14$$

This acts at a distance of 4.8' from the bottom.

The total load against the abutment is 347.295
1145.82
11933.14

3426.255 tons.

To find the point of application of the total load of
6.255 tons.

$$3426.255\,X = 1933.14\,(4.8) + 1145.82\,(14.55) + 347.295\,(23\text{-}1/2)$$

$X = 9.94'$ from the base.

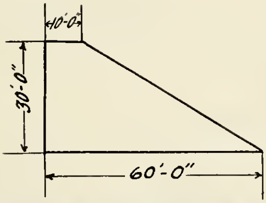
The width of abutment most suitable to this dam seemed to be
feet. The longitudinal cross-section is as shown

The weight of the abutments

750 x 10.0 = 7500 cu.ft.

Total = 26250 cu.ft.

= 1969 Tons.



Combining these forces graphically we find that the resultant
t passes on the edge of the middle third which shows that the
tment is safe against overturning.

entre Abutment:

The total pressure of water behind the dam must be aken up by the two end abutments, and the centre abutment. The centre)utment takes half of total pressure, and equals height of dam x wt. f cu.ft. of water x 1/2 C, when C is straight distance between end)utments.

or $P = h \times w \times 1/2c$
when P = total pressure.

But since chord lengths vary, the height will be divided into 12 ections of 10 ft. each. This will make the height of each segment easured on the inclined plane $= \frac{10}{\cos 30^\circ} = \frac{10}{.866} = 11.55$ ft.

Length of 1st segment from top = C =	312 ft.	$\frac{C}{2}$ = 156
2nd	312	156
3rd	312	156
4th	312	156
5th	250	125
6th	240	120
7th	220	110
8th	195	98
9th	162	81
10th	140	70
11th	104	52
12th	104	52

ressure on any segment,

$$= \quad P_1 = \tfrac{1}{2}(h_1 + h_2) \times h \times W \times \tfrac{c}{2}$$

here h_1 = head on top of segment
h_2 = head on bottom of segment
h - height of segment.

st: Pressure on first segment, since it is vertical is

$$P_1 = \tfrac{1}{2} wh^2 \times \tfrac{c}{2} = \tfrac{1}{2} \times 62.5 \times \overline{10}^2 \times 156 = 487,500^{\#}$$

Pt. of application is $\frac{10}{3} = 3\frac{1}{3}$ ft.from bottom of division.

nd. Pressure on 2nd segment $= P_2 = \frac{1}{2}(10+20) \times 62.5 \times 11.55 \times 156 = 1,690,000^{\#}$

Find the pt.of application, which is the c.g. of pressure trapezoid by graphics.

This is 4.4 ft. from bottom of segment.

rd: $P_3 = \frac{1}{2}(20 + 30) \times 62.5 \times 11.55 \times 156 = 2,800,000\#$

pt.of application 4.7 ft. from bottom division.

Note: The distances of the pts. of application of forces are easured on the vertical, and give the distance up from the bottom ivision of segment to where force is considered as applied.

th: P_4 = ½(30 + 40) x 62.5 x 11.55 x 156 = 3,920,000#
pt. of application 4.7 ft. above bottom division.

th: P_5 = ½(40 + 50) x 62.5 x 11.55 x 125 = 4,050,000#
pt. of application 4.7 ft. up from bottom division.

th: P_6 = ½(50 + 60) x 62.5 x 11.55 x 120 = 4,780,000#

th: P_7 = ½(60 + 70) x 62.5 x 11.55 x 110 = 5,150,000#
pt. of application 4.8 ft. from bottom division.

th: P_8 = ½(70 + 80) x 62.5 x 11.55 x 98 = 5,300,000#
pt. of application 4.8 ft. above bottom division.

th: P_9 = ½(80 + 90) x 62.5 x 11.55 x 81 = 4,975,000#
pt. of application 4.8 ft. above bottom division.

0th: P_{10} = ½(90 + 100) x 62.5 x 11.55 x 70 = 4,800,000#
pt. of application 4.9 ft. from bottom division.

1th: P_{11} = ½(100 + 110) x 62.5 x 11.55 x 52 = 3,920,000#
pt. of application 4.9 ft. above bottom division.

2th: P_{12} = ½(110 + 120) x 62.5 x 11.55 x 52 = 4,320,000#
pt. of application 4.9 ft. above bottom division.

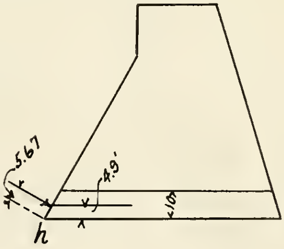
Total sum of all forces acting on inclined part of dam = 45705000#

To find the point of application of the resultant of all forces,
ay off each individual force
n exact scale and find lever arm.
ake moments about heel h

$\frac{4.9}{\cos 30} = \frac{4.9}{.866} = 5.67$ ft.

$\frac{4.8}{.866} = 5.54'$, $\frac{4.7}{.866} = 5.44'$

$\frac{4.4}{.866} = 5.08'$



= (4,320,000 x 5.67 + 3,920,000 x 17.22 + 4,800,000 x 28.77 +
4,975,000 x 40.19 + 5,300,000 x 51.74 + 5,150,000 x 63.29 +
4,780,000 x 74.74 + 4,050,000 x 86.29 + 3,920,000 x 97.84 +
2,800,000 x 109.39 + 1,690,000 x 120.58) ÷ 45,705,000

= 57.6 ft. measured on incline.

Now combine this resultant (See Fig. 1, Drg. # 2) with the orizontal force acting on top 10 ft. and find resultant fpcee and irection.

This is found to be 46,000,000#

To find whereresultant of all forces acting cuts the base ume following dimensions for abutment.

12' wide on top and 6' thick
End *batter* 6 to 12
Side ·· 1 in 10
ure weight and volume by pris~~a~~moidal formula.

wt. = $\frac{l}{6}(A_1 + 4A + A_2) \times 150$
l = height = 120
A_2 = Area of top
A_1 = Area of bottom
A = Center area
150 = wt of cu. ft. of conrete

∵ wt = $\frac{120}{6}$(135 x 30 + 4 x 75 x 18 + 6 x 12)150 = 28,242,000#

Find the cg. by graphics. To find wt.of wall resting on abut-
t. Assume radius of curvature as 100'.
Then $T = R\frac{P}{S}$

T = Thickness of wall
R = Radius of upstream face.
P = Water pressure / sq. ft.
S = Allowable unit stress = 350# sq.in.

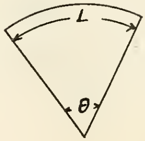
This is same formula as used in figuring pipes.
The maximum in arch rib will be at the bottom.

P = 62.5 x 120 = 7500 lbs. per sq.ft.
T = $\frac{100}{350} \times \frac{7500}{144}$ = 14.9 ft.make it 16' even.

Make arch rib 3' thick on top and 16' on bottom.
e: - The thickness on top will be assumed to be sufficient to resist pressure.

Since the thickness varies uniformly with the pressure, the ckness of wall will vary uniformly between 3' on top and 16' on tom. To get wt. of wall it will be necessary to get length of arch).

L = θ R
L = Length
θ = Angle *subtended* at center.
R = Radius of arc = 100'



Length of Chord.	Angle at Centre θ	Length of Arc.
156	102.0°	173 ft.
125	76.5°	134 ft
120	73.25°	128 ft
110	66.5 °	116 ft
98	58.0 °	101 ft
81	47.0°	82 ft
70	40.0°	70 ft
52	30.0°	52 ft

But only a part of weight of arch rib goes into abutment. o get at this, draw to scale a plan of dam.

See Fig. 2, Drawing #(2)
Everything above the vertical plane $V V'$ will pass into butment. That below passes directly into base.

Divide inside half of each arch into eight strips as shown. idth of all but No. 8 are 10 ft.

No 8 is 8 ft. wide.
Then calculate weight of each strip. The actual length will e the length measured on the drawing as a projection, divided by as 60 degrees. Since dam is inclined 60° to horizontal,

f y = actual length
x = measured length
hen y = x/cos 60 = $\frac{x}{1/2}$ = 2x

or actual length is just twice measured length.

Section No.	Actual length x AV. Thick x width x 150		wt.
1	57 x 2 x 1/2(3 + 16)x 10 x 150	=	1,630,000
2	48 x 2 x 1/2(3 + 14.6) x 10 x 150	=	1,260,000
3	41 x 2 x 1/2(3 + 13) x 10 x 150	=	985,000
4	36 x 2 x 1/2(3 + 12) x 10 x 150	=	810,000
5	31 x 2 x 1/2(3 + 10) x 10 x 150	=	610,000
6	30 x 2 x 1/2(3 + 9.3)x 10 x 150	=	520,000
7	28 x 2 x 1/2(3 + 7.9)x 10 x 150	=	427,000
8	27 x 2 x 1/2(3 + 6.6)x 8 x 150	=	316,000
	wt.of 1/2 of 1 arch		6,559,000#

Total weight of both arches going into ~~sheet~~ abut. =13,118,000#

To find the distance of c.g. of this mass from pt.of heel. Find the c.g of each strip by graphics as shown in Fig. 5, Drawing No. 2 Takemoments about heel as before

$$x = \frac{(1,630,000 \times 22 + 1,260,000 \times 18 + 985,000 \times 16 + 810,000 \times 15 + 610,000 \times 14 + 520,000 \times 13 + 427,000 \times 12 + 316,000 \times 11)}{6,559,000\#}$$

$$.. \; x = \frac{110,393,000}{6559000} = 16,8 \text{ ft.}$$

That is, the mass acts 16.8 ft to right of h.

Now combine this weight with the weight of the abutment and then with the thrust of the water and find resultant.

This is 75,500,000# and falls well within the middle third.

Stress on base on a plane perpendicular to line of thrust.

$$\frac{75,500,000}{115 \times 30 \times 144} = 152\#/\text{sq.in.} = \text{Direct Thrust.}$$

$$S = \frac{M}{I/e} = \frac{75,500,000 \times 8}{5 \times 115^2 \times 144} = 64\#/\text{sq.in. Stress due to bend. M.}$$

$152 + 64 = 216\#$ max stress on toe

$152 \mp 64 = 88\#$ stress on heel.

Shear. Horizontal component of total thrust producing shear is 2,000,000#.

$$\frac{2,000,000}{135 \times 30 \times 144} = 4\# \text{ safe}$$

Now test for the bottom of 7th section. Weight of abutment above this plane is

$$\frac{70}{6}(81 \times 20 + 4 \times 46.5 \times 13 + 6 \times 12)\,150 = 7,193,000\# \qquad \textit{See Fig 3-D-2}$$

Consider none of wall weight to go into this part of abutment. As a matter of fact some of it does go, but it is on the side of safety to neglect it.

To find resultant water pressure:

Find the pressure and the point of application on the inclined part. Take moments about h, same method as before.

$$x = (5,150,000 \times 5.67 + 4,780,000 \times 17.14 + 4,050,000 \times 28.69 + 3,920,000 \times 40.24 + 2,800,000 \times 51.79 + 1,690,000 \times 62.98) / 22,740,000$$

$$\therefore x = \frac{636,100,000}{22,740,000} = 28'$$

That is the pressure acts 28 ft. (measured on incline) above heel
Now combine this with the horizontal pressure on the top 10 ft.
l combine this resultant with weight of abutment. Final resultant
.ls just within middle third.

Since some weight of arch is on there this is perfectly safe.

Stress on plane perpendicular to thrust.
Resultant thrust is 27,000,000#

$$\frac{27,000,000}{56 \times 20 \times 144} = 167\# \text{ direct thrust}$$

$$S = \frac{M}{I/e} = \frac{27,000,000 \times 9}{\frac{1}{6} \times 20 \times 56^2 \times 144} = 161\#$$

Total stress = 167 + 161 = 328#/sq.in. in toe
167 - 161 = 6# in in heel

O.K.

ar:

Horizontal force producing shear is 10,000,000#. Arch at
s point is 10.8 ft. thick. Let it take its proportionate amount
shear.

$$\frac{10,000,000}{(81 \times 10.5) \times 20 \times 144} = 38\#$$ Safe since arch on either side of abutment takes some shear.

IGN OF END ABUTMENT. Height 30 ft.

To find thrust at 30' depth.

T = RP = 100 x 62.5 x 30 = 187,500# per sq. ft.
On top it is zero.

$$\text{Total thrust } \frac{187,500}{2} \times 30 = 2,800,000\#$$

Point of application 10 ft. above base.

Assume the dimensions thus: *See Dr. 5*
Since this connects with spillway section which is vertical. The thrust will be assumed as acting horizontally.

Weight of abutment

$$= \frac{30}{6}(25 \times 12 + 4 \times 15 \times 37\text{-}1/2 + 18 \times 50) = 2,587,500\ \#$$

Combine this weight with thrust. Resultant falls well within idle third.

4,400,000# = Resultant.

$\frac{4,400,000}{34 \times 18 \times 144}$ = 50#/sq.in. direct stress on plane perpendicular to thrust.

Stress due to bending.

$S = \frac{M}{I/e} = \frac{4,400,000 \times 4}{\frac{1}{6} \times 18 \times \overline{34}^2} = 35\#$

50 + 35 = 85#/sq.in. Stress in toe.
50 - 35 = 15#/sq.in. " " heel

$\text{Shear} = \frac{300,000}{18 \times 50 \times 144} = 7\#$

O.K.

DAGE:

Since the weight has already been found of the abutments, iding by 27 x 150 will give the yardage.

Centre abutment = 28,242,000# = 28,242,000

2-end-abutments, at 2,587,000# each = 5,174,000
Total = 33,416,000#

$\frac{33,416,000}{27 \times 150}$ = 8,200 yds.

DAGE IN GRAVITY DAM:

The elevation of dam was subdivided into strips of varying thickness, t, to correspond with the steps cut into the rock profile. Then from crosseetion view of dam the area corresponding to the depth given in table below was found. This multiplied by thickness , t, gives volume.

pth-d	Thickness-t	Area of Section	Val.in cu.ft.
36	20	550	11,000
42	20	695	13,900
52	10	988	9,880
61	10	1312	13,120
73	15	1839	27,600
81	10	2224	22,240
0	10	2722	27,220
95	10	3022	30,220
15	20	4402	88,040
24	85	5100	425,000
95	26	3022	78,600
86	12	2494	29,800
76	8	1971	15,768
71	12	1747	21,000
63	15	1391	21,000
53	10	1040	10,400
43	10	722	7,220
40	10	645	6,450

Total $\frac{858458}{27}$ 31,800 cu.yds. concrete.

MPARISON:

Volume of gravity dam 31,800 cu.yds.
Volume of single arch dam 19,900 cu. yds.
Volume of double arch dam 18,420 cu.yds.

In the light of the present investigation, we are able to rm an approximate idea of the relative economic values of the three fferent types of dam, namely, the gravity dam, the single arch dam d the double arch dam. From the above figure we can say that the uble arch dam and the single arch require about 60% as much material the gravity section.

Between the single arch and the double arch for the conditions hand, the relative advantages and disadvantages are about the same. e single arch requires greater length for the same distance between utment, greater cross-section for the same head of water on account its greater radius of curvature and stronger and heavier abutments on count of the greater thrust. The double arch however requires a ntre abutment which for great heads becomes a very important factor the total volume.

In a case where the banks were of durable rock formation and se abruptly, the single arch would be preferable, as no abutments, sides the natural cliffs would be necessary.

Where the head of water to be stored was not as large and tificial abutments, would be necessary the double arch would have l the arguments in its favor. Where the side hills were unable

o take any thrust on account of the character of the material, the idea f the arch dam would be entirely discarded and the gravity type used.

From the foregoing conclusions, we would therefore say that he type of dam to be used depended entirely upon geologicaland topo- raphical conditions, and while an arch dam would be used in some cases ith the greatest economy, it would in others be condemned as the rossest waste.

TABLE 1.

CROSS SECTION OF ARCH DAM.

HEAD on Section.	Theoretical Width.	Actual Width.
0 Feet	0 Feet	6.0 Feet.
10	2.75	7.7
20	5.50	9.2
30	8.25	11.0
40	11.00	13.0
50	13.75	15.0
60	16.50	17.4
70	18.55	20.0
80	22.00	22.3
90	24.75	25.0
100	27.50	28.4
110	30.25	32.0
120	33.00	36.7
132	36.30	42.0

Front Face of Dam Vertical

Back of Dam, form of Parabola whose equation is $y^2 = \frac{44}{3} x$.

I. L. Sapczycki
H. H. Jack

TABLE 2.

VOLUME OF ARCH DAM.

AVERAGE WIDTH OF SEC.	SECTION AREA.	VOLUME OF SECTION.
9 Feet	126.5 Sq Ft.	1136.25 Cu. Ft.
29	413.9	11980
15	576.0	8640
21	747.6	15700
9.5	883.0	8380
10	1008.6	10086.0
15.5	1237.25	19190
31	1440	44600
12.5	2167.4	27000
12.5	2315.25	28900
17	2634.5	44,800
12	2315.25	27750
20	2203.9	44078
16	2167.4	34700
24	2097.4	50300
12	1467	17604
12.5	1336.2	16700
12.5	1120	14000
18.5	934.6	18200
14	707	9900
14	580	8120
29	413.9	11980
32.5	3236	10500

Volume of Dam in place 31,800 YDS.
Volume of Arch Dam 19,900 YDS.

I. L. Szeszycki
H. H. Beck

TABLE 3.

CROSS SECTIONS AT 10 Ft. INTERVALS.

Number of Section	Height of Section	Area of Section	Summation of Sections.
1	10	68.5	68.5
2	10	84.5	153.0
3	10	101.0	234.0
4	10	120.0	374.0
5	10	140.0	514.0
6	10	162.0	676.0
7	10	187.0	863.0
8	10	211.5	1074.5
9	10	236.5	1311.0
10	10	284.0	1595.0
11	10	302.0	1897.0
12	10	343.75	2240.75
13	12	393.75	2634.5

Areas figured as trapezoids.

Total Area of Section. 2634.5 Sq Ft.

I. L. Szczycki
R. R. Pack

TABLE 4.

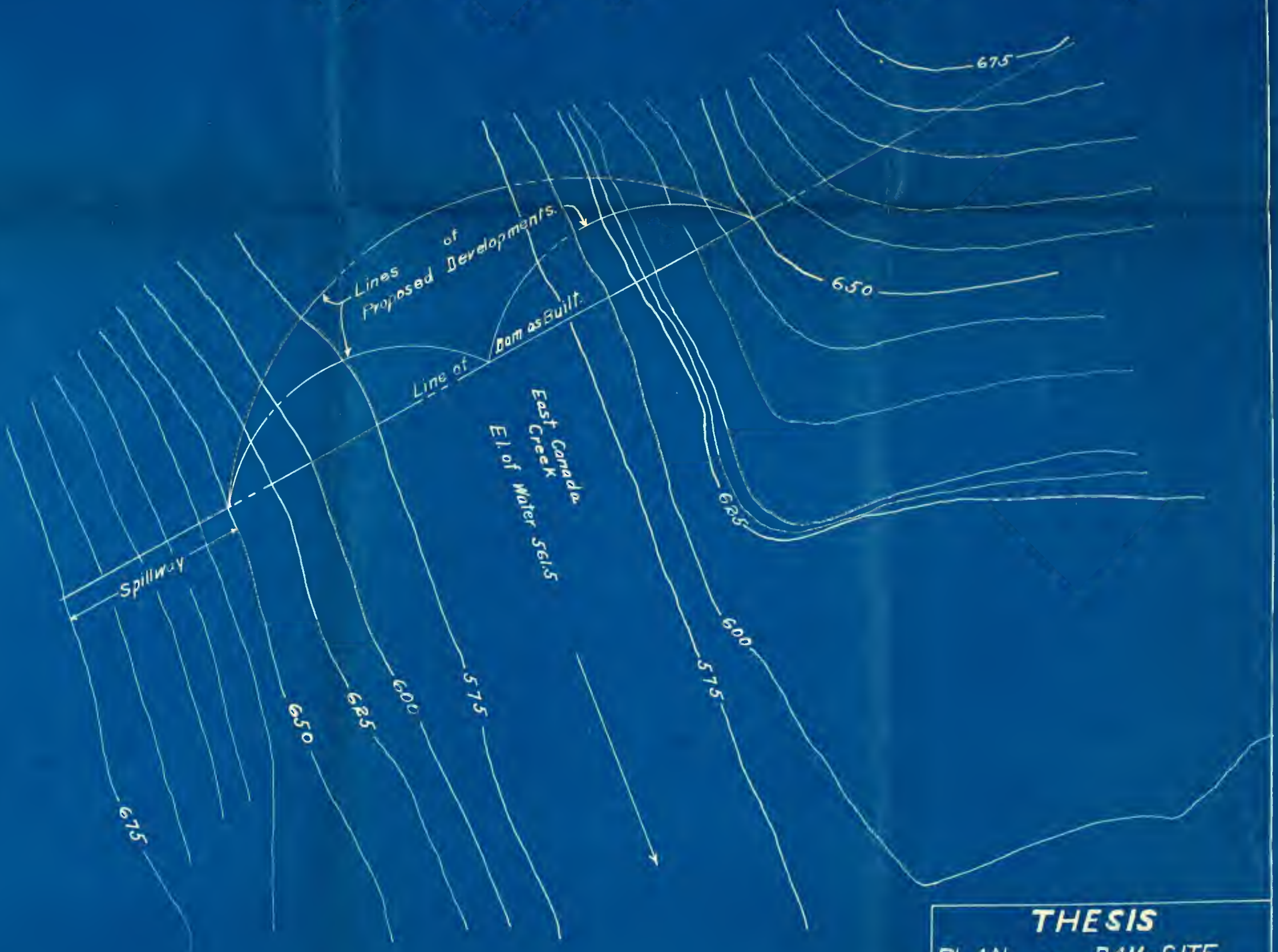
VOLUME OF HOLLOW ARCH DAM.

Width of Section = 11.55'

No. of Segment	Thickness	Length.	Yardage
1	3	173	222
2	3	173	222
3	4.3	173	320
4	5.6	173	416
5	6.9	134	396
6	8.2	128	450
7	9.5	116	470
8	10.8	101	467
9	12.1	82	425
10	13.4	70	402
11	14.6	52	324
12	160	52	356
Σ V			4770

Volume of one Arch	4470 Yds.
" " both Arches	8940 "
" " Abutments	8200 "
" " Trench	1280 "
Total	18,420 "

I. L. Szczygcki
R. H. Pack

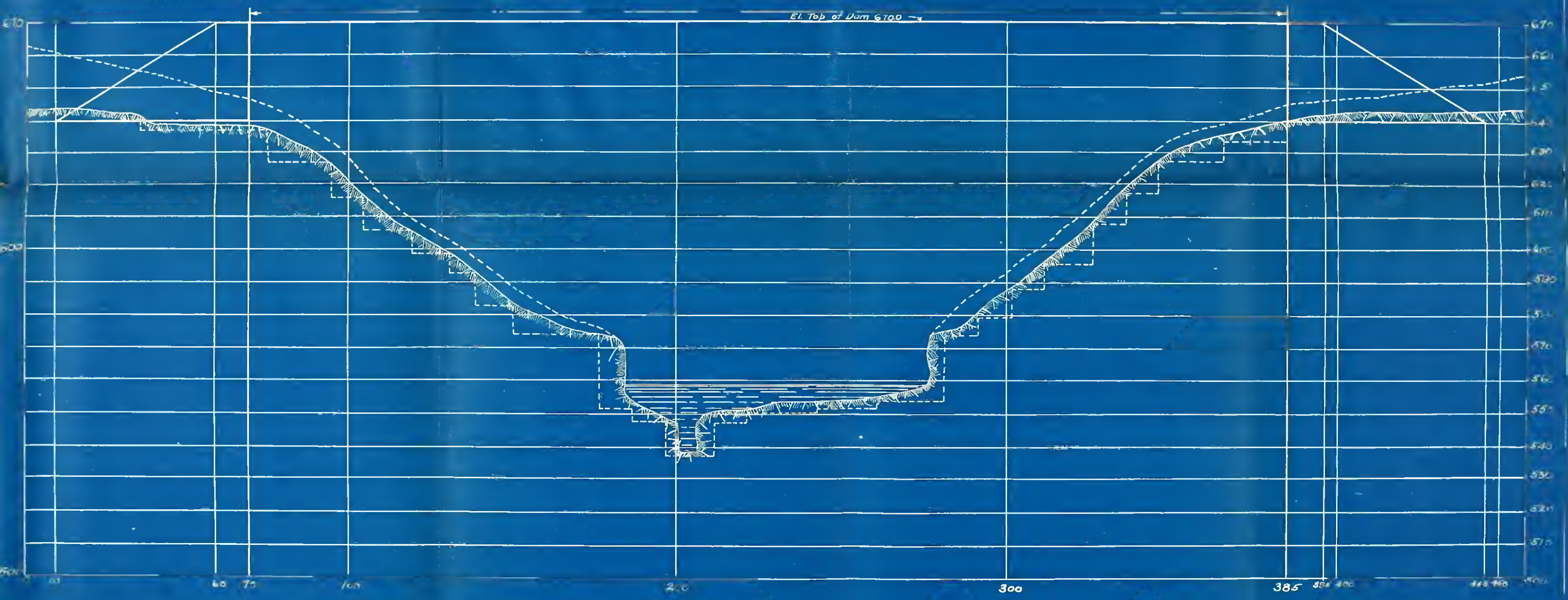


THESIS

PLAN OF DAM-SITE
EAST CREEK ELEC. CO.
SCALE:-
NOV. 15, 1910

W. R. Zack. J. L. Szaszycki

Drwg # 1

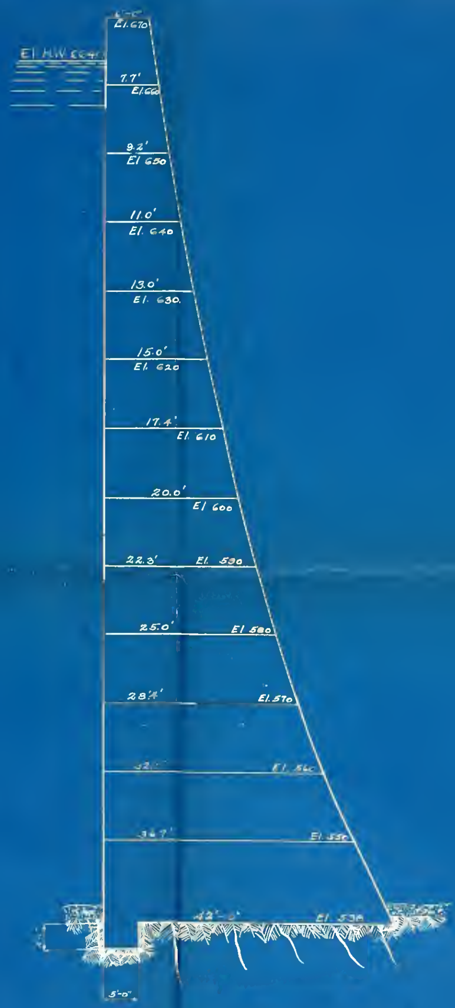


THESIS

Elevation of Dam Site
EAST CREEK ELEC. CO.
SCALE 20'-1" FEB 23 1911.

J L Szyzyki N. H. Gack

Drg No.
1 A



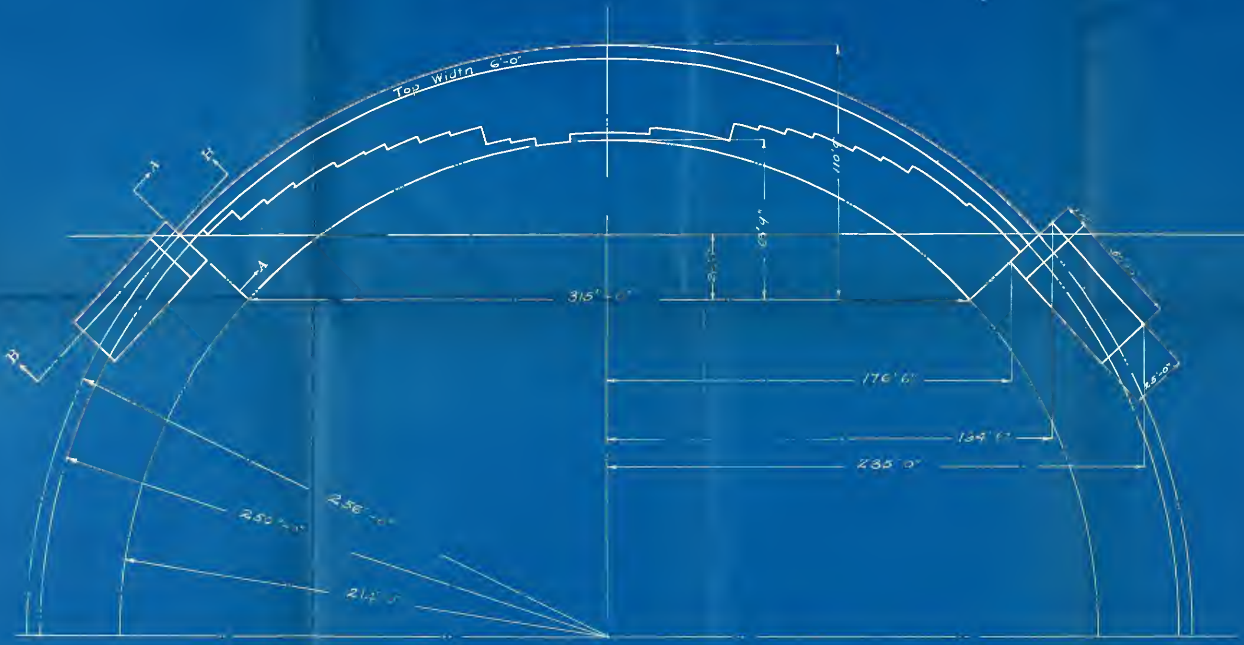
THESIS

Profile of Arched Dam.

EAST CREEK ELEC. CO.

SCALE 10'=1" FEB 27. 1911.

J.L. Szczepski R.K. Zack Drg No. 3A



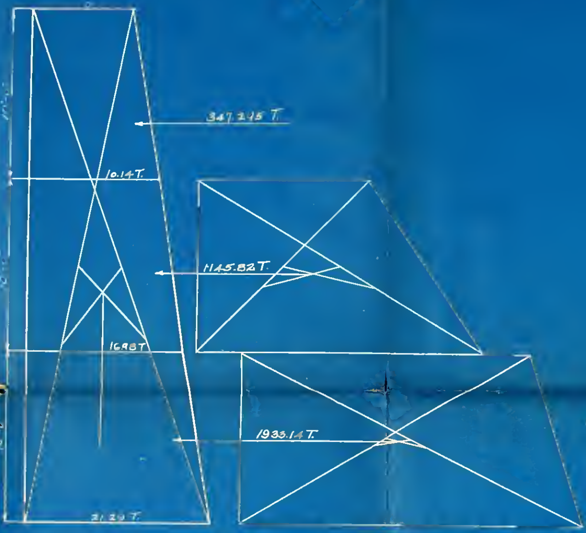
THESIS

Plan of Arched Dam.

EAST CREEK ELEC. CO.

SCALE 30'=1" FEB 2 1911.

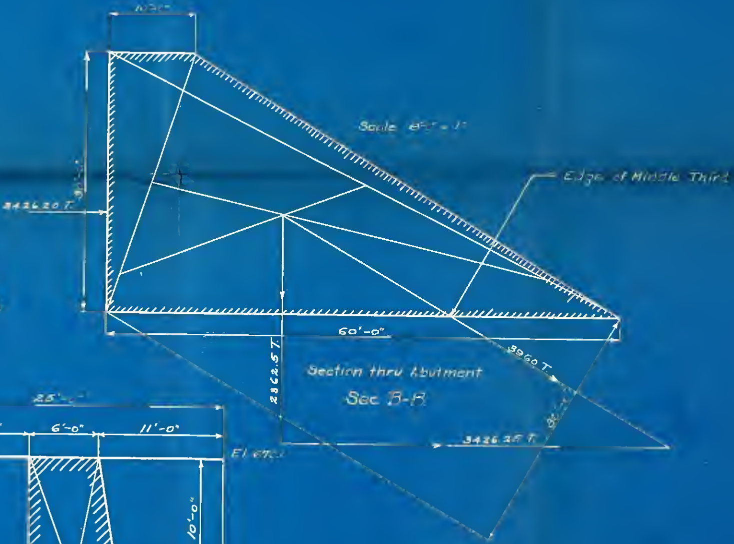
I L Szczygycki F.K. Jack Drg No 3 A



Scale 4 Ft = 1"

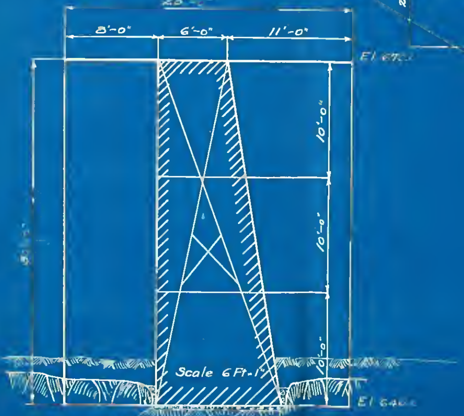
Determination of Resultant Pressure on Abutment due to Stress in Arch

T = 1 Ton = 2000 lbs.



Section thru Abutment

Sec B-B



Plan showing Location of Dam with respect to Abutment. Sec A-A

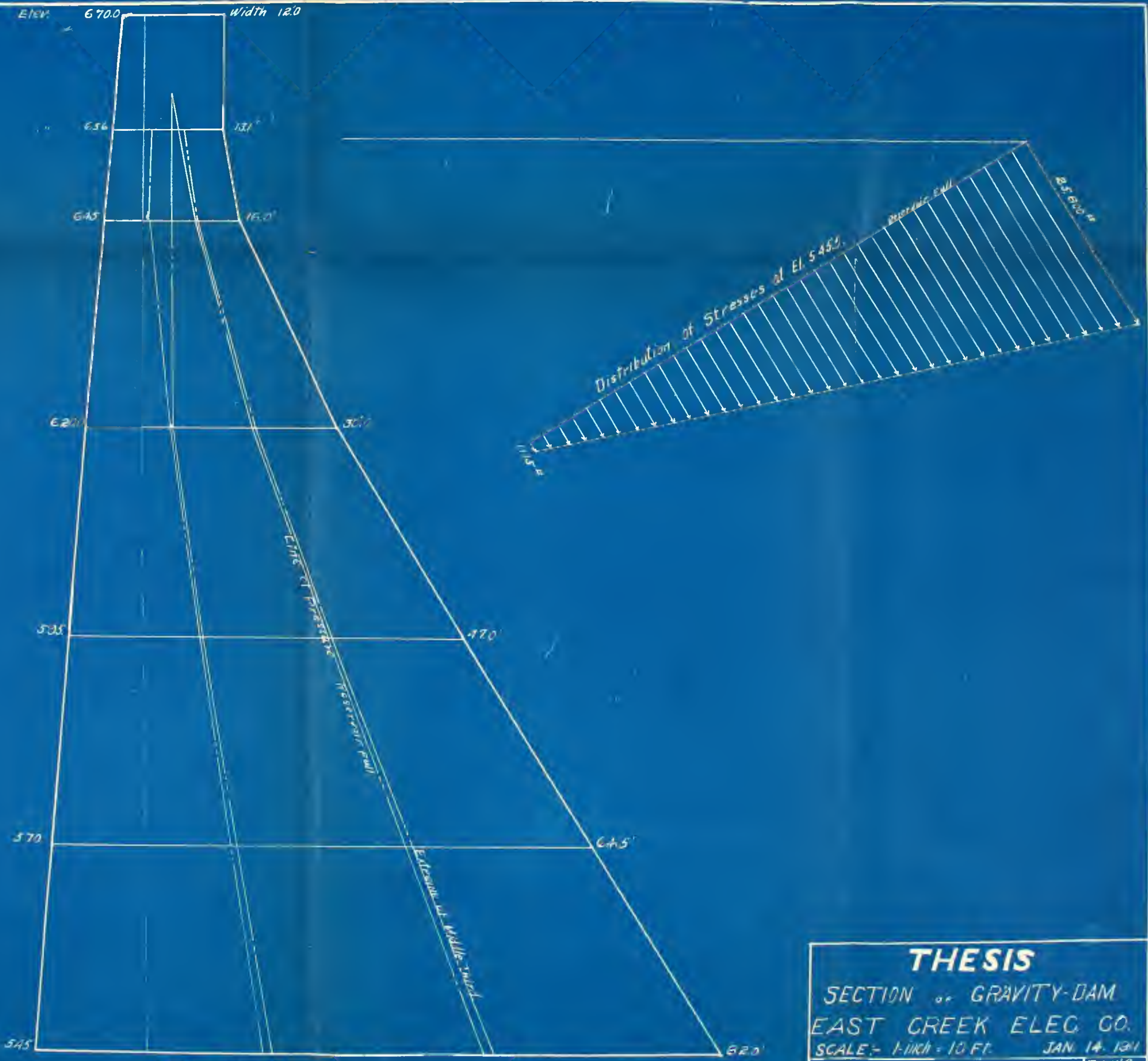
THESIS

Design of Abutments.

EAST CREEK ELEC. CO.

SCALE as shown. Mar 10. 1911.

Drg No. 4 A



Factor of Safety Against Overturning 2.1

THESIS

SECTION of GRAVITY-DAM
EAST CREEK ELEC CO.
SCALE:- 1 inch = 10 Ft.   JAN. 14. 1911

R. R. Zack   I. L. Szczepski   Drw'g. # 1

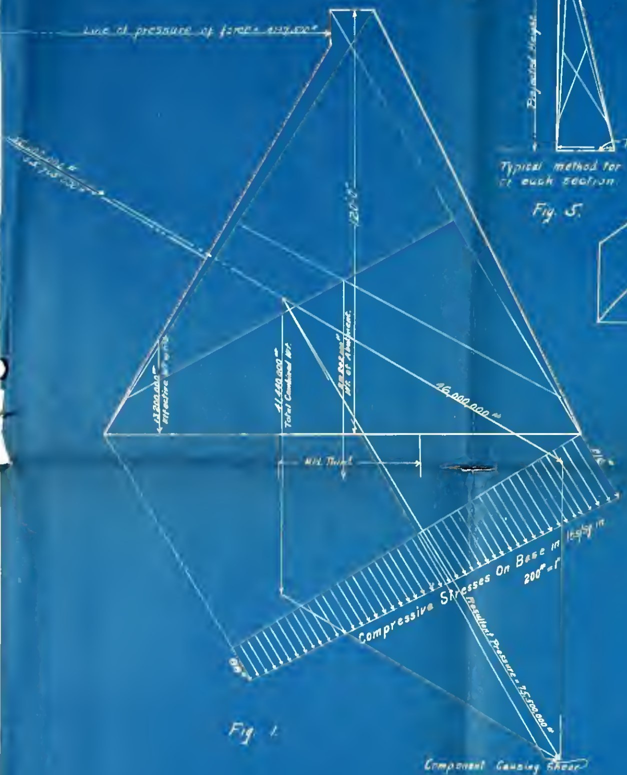
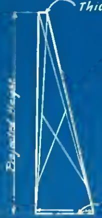
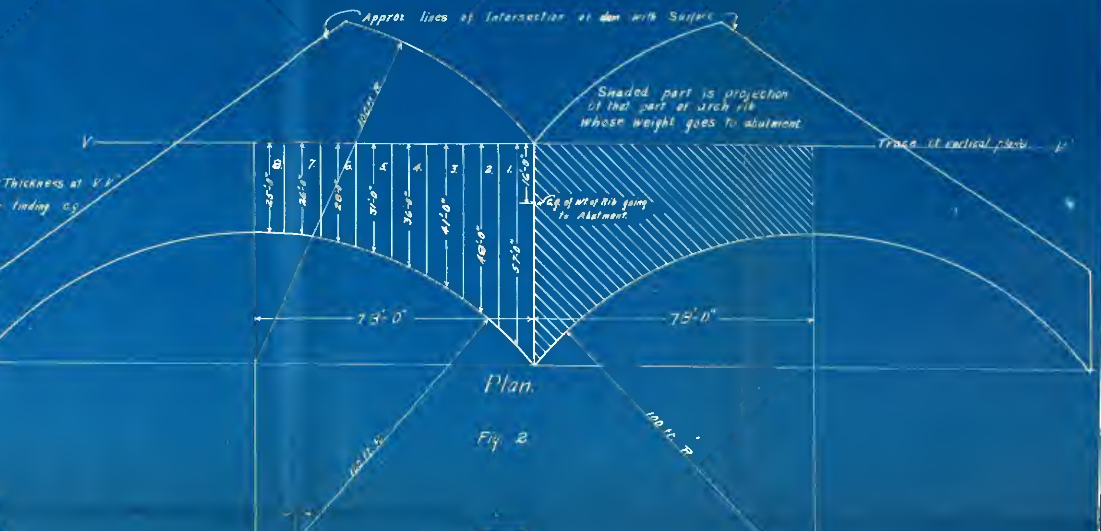

Line of pressure of force = 457,500#
120'-0"
13,200,000# Effective Wt. of Rib
41,440,000# Total Combined Wt.
28,240,000# Wt. of Abutment
16,000,000#
Mid. Third
Compressive Stresses On Base in lbs./sq. in. 200# = 1"
Resultant Pressure = 75,500,000#
Component Causing Shear

Fig. 1.



Typical method for finding c.g. of each section.

Fig. 5.


Approx. lines of Intersection of dam with Surface
Shaded part is projection of that part of arch rib whose weight goes to abutment
V
Trace of vertical plane V
8.
7.
6.
5.
4.
3.
2.
1.
35'-0"
26'-0"
20'-0"
31'-0"
36'-0"
41'-0"
49'-0"
57'-0"
16'-0"
C.G. of Wt. of Rib going to Abutment
78'-0"
78'-0"
100 ft. R
100 ft. R
100 ft. R

Plan.

Fig. 2.

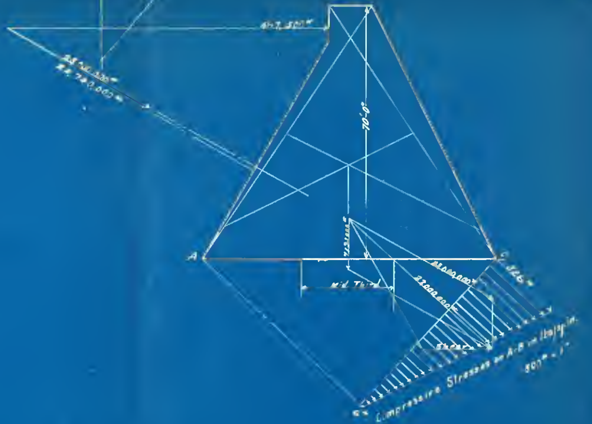


Fig. 3.

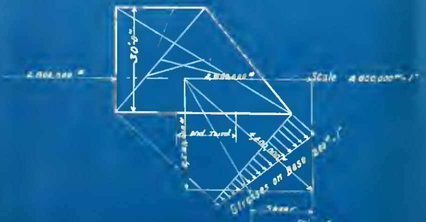


Fig. 4.

THESIS

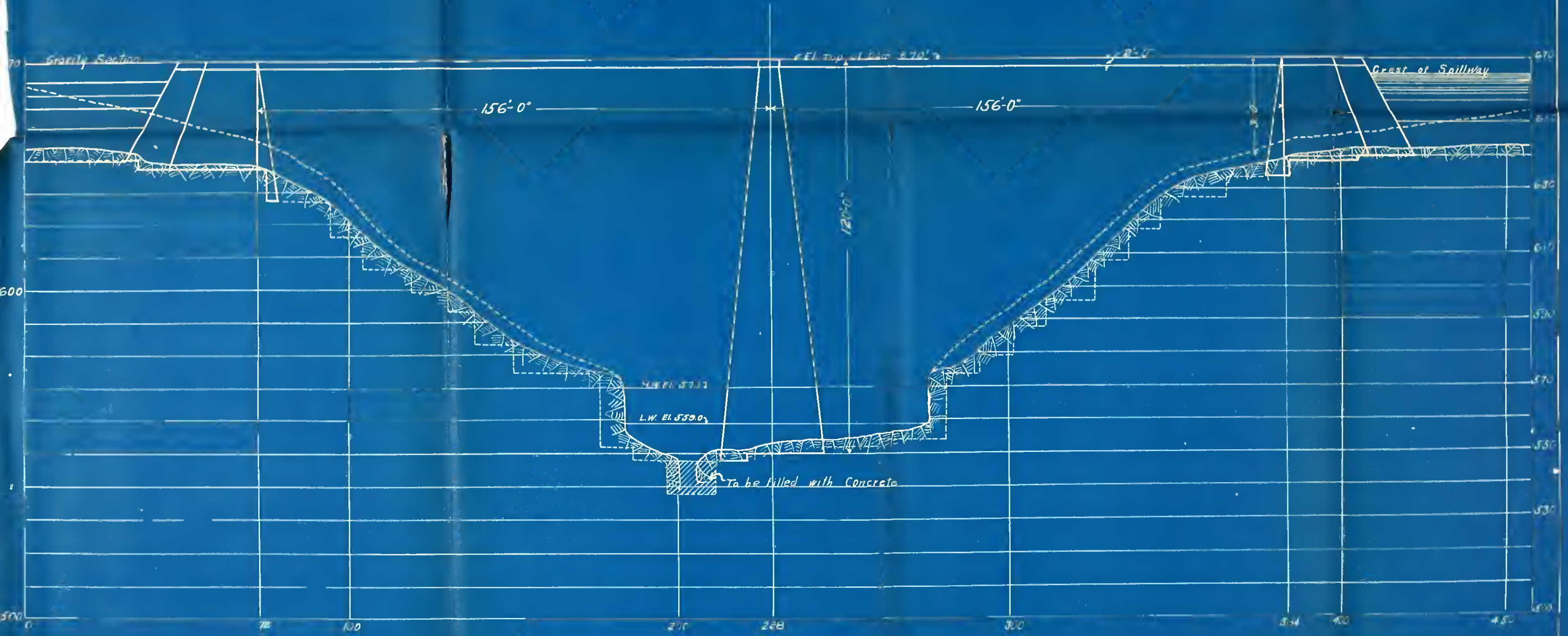
ANALYSIS OF STRESSES

EAST CREEK ELEC. CO.

SCALE :- 1" = 20'-0"
1" = 10,000,000#

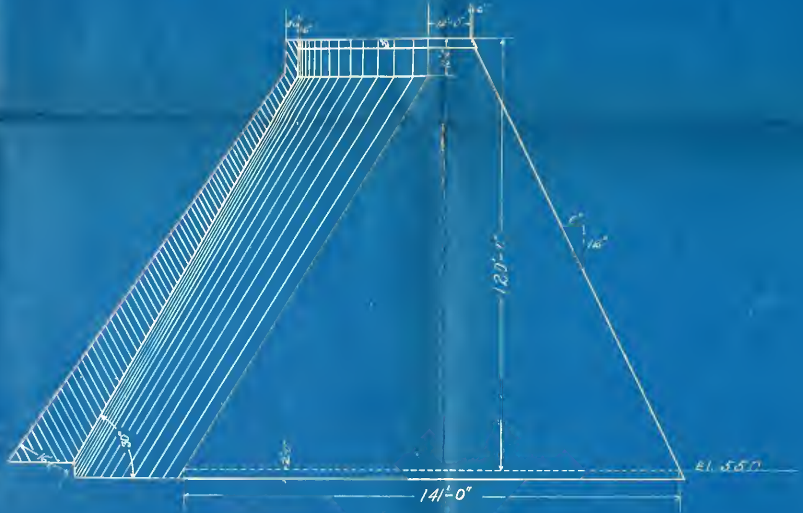
JAN. 23, 1911.

Drwg. # 2

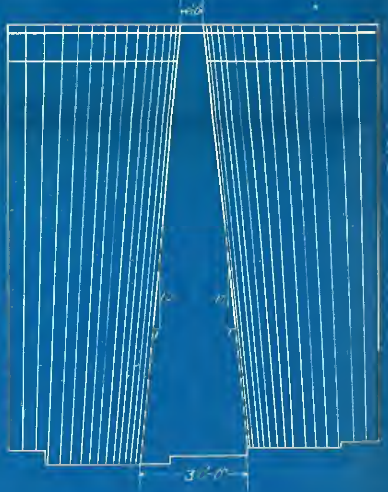


THESIS

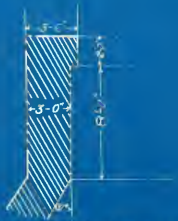
DOWN STREAM ELEVATION OF DAM
EAST CREEK ELEC. CO.
SCALE:- 1"=20'-0"
JAN. 20, 1911
Drwg



Section at A-B



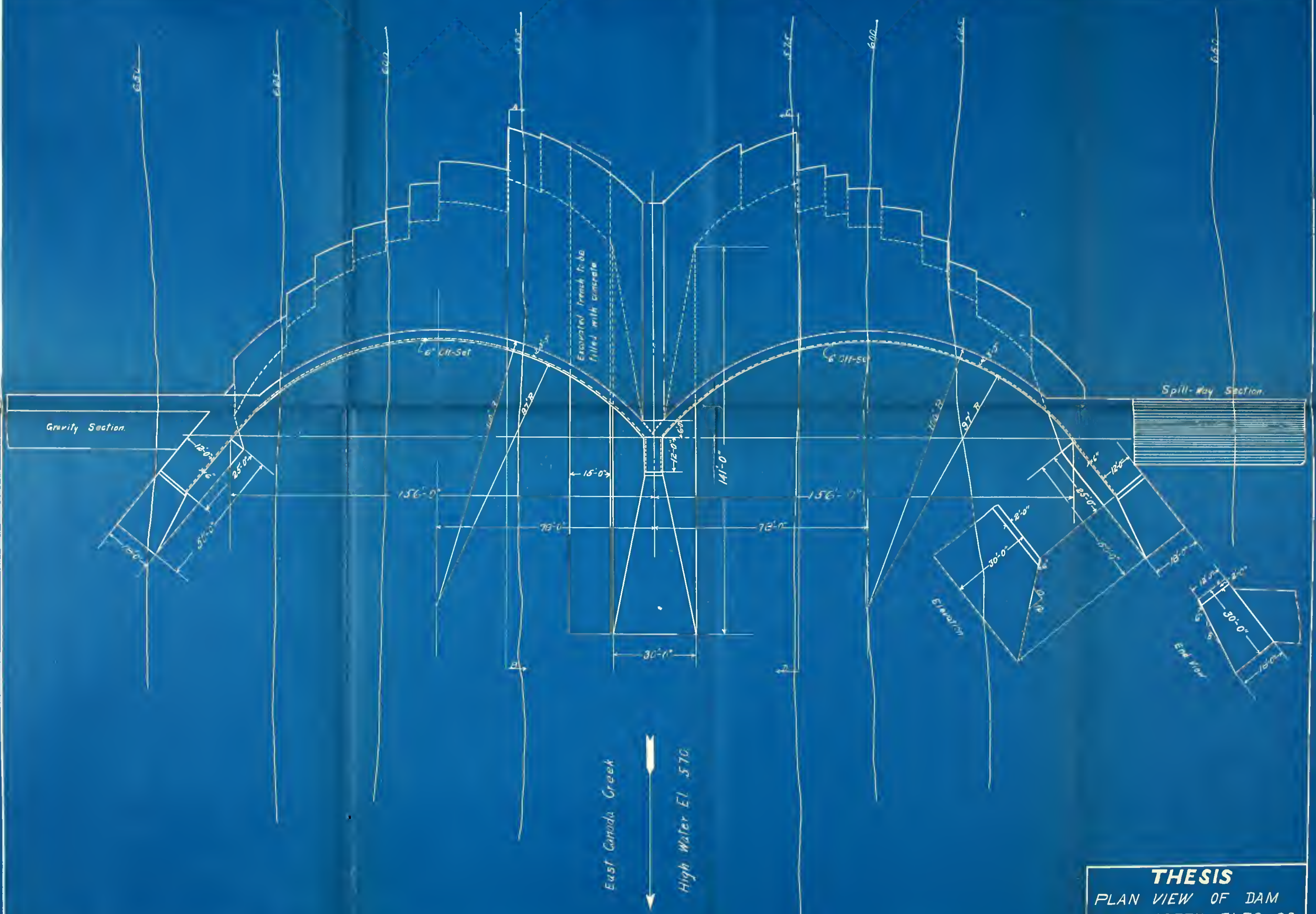
End View Between Sections AB & CD



Section through vertical part of rib showing coping. Scale 1"=5'-0"

THESIS

DETAILS OF CENTRE ABUTMENT.
EAST CREEK ELEC. CO.
SCALE:- 1"=20'-0"
JAN. 19, 1911
R.R. Jack I.L. Szczycki
DRW'G # 4



THESIS

PLAN VIEW OF DAM
EAST CREEK ELEC. CO.
SCALE:- 1" = 20'-0"
JAN. 21, 1911.
DRW'G # 5